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A comparison of current analytical methods for predicting soil-structure interaction due to tunnelling

Giorgia Giardina^{*1}, Matthew J. DeJong², Benjamin Chalmers², Bryan Ormond², Robert J. Mair²

Abstract

Current procedures for the assessment of buildings response to tunnelling take into account the effect of soil-structure interaction through the definition of the building stiffness relative to the soil stiffness. Limitations of these procedures are uncertainties in the evaluation of structural parameters and inconsistent results between different methods. In this paper, three existing formulations of the Relative Stiffness Method (RSM) were been critically evaluated by analysing the governing factors in the building stiffness calculation and their effect on the structural damage assessment. The results of a sensitivity study on building height, eccentricity, opening ratio, tunnel depth, soil and masonry stiffness, and trough width parameter quantified the effect of these factors on the considered RSMs. The application of different RSMs to a real masonry building adjacent to the Jubilee Line tunnel excavation highlighted the significant effect of window openings, façade stiffness and neutral axis position on the building stiffness calculation and deformation prediction. These results highlight the need for a consistent and robust damage assessment procedure.

Keywords: building damage, building stiffness, masonry structures, relative stiffness, soil settlements, soil-structure interaction

1. Introduction

Underground constructions in urban areas require monitoring and protection of surface buildings. For large scale projects, like the North-South Line in Amsterdam, the Jubilee Line Extension and Crossrail in London, the costs connected to the preliminary damage assessment of structures can represent a large portion of the total project investment [1]. Damage prediction procedures are required to screen a large number of buildings in a relatively short time frame; furthermore, they need to be conservative, but accurate enough to avoid unnecessary further analyses.

Current methods applied to large projects involve a phased procedure where analyses of increasing complexity are applied to progressively smaller groups of buildings [2]. While offering the significant advantage of a rapid and extensive assessment, such a procedure has the limitation of not taking into account potentially relevant components

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of the building response until the very final phase. In particular, it neglects the interaction between the building and the excavation-induced settlement trough when predicting the deformations of the soil-structure system.

More detailed methods exist which take this interaction into account [3–6]. These methods are based on the assumption that the main component affecting the soil-structure interaction is the relative stiffness between the building and the soil. However, these methods are not widely adopted in practice, due to concerns about the uncertainties in the base assumptions and their effects on the final damage prediction. Major issues involve the calculation of the global stiffness of the building under assessment, and disagreement regarding how to normalise the building stiffness with the soil stiffness to produce a relative stiffness.

An important step towards the definition of a more robust damage assessment procedure consists of a better understanding of the uncertainties and discrepancies in the available methods. This paper critically analyses the existing relative stiffness methods for the assessment of excavation-induced damage to buildings. In particular, it focuses on the evaluation of the global building stiffness, by investigating (a) what are the governing structural features influencing the building stiffness calculation and (b) how uncertainties and assumptions in the definition of these factors affect the final damage assessment.

In the following sections, a detailed summary of the available methods for the assessment of settlement-induced damage is first presented, with a specific focus on procedures for calculating both the absolute and relative building stiffness. A sensitivity study is then presented to broadly highlight the effect of buildings stiffness assumptions on the final damage assessment. Subsequently, a case study is considered to more specifically quantify the effect of various assumptions on damage assessment. Finally, the conclusions identify reasons for inconsistencies between the current procedures and suggest opportunities towards the formulation of an improved method.

2. Literature review

2.1. Limiting Tensile Strain Methods (LTSM)

2.1.1. Bending-based strain criterion

This empirical-analytical method for the assessment of structural damage due to ground deformations was originally formulated by Burland et al. [7] and further developed by Boscardin and Cording [8]. First, the soil movements due to the underground excavation are calculated without taking into account any interaction with adjacent structures. These so-called greenfield displacements are then imposed to a simplified beam model of the building. Based on Timoshenko beam theory [9], the maximum bending strain $\varepsilon_{b,\max}$ and the maximum diagonal strain $\varepsilon_{d,\max}$ are derived as:

$$\varepsilon_{b,\max} = \frac{\Delta/L}{\left(\frac{L}{12t} + \frac{3I}{2tLH} \frac{E}{G}\right)} \quad (1)$$

$$\varepsilon_{d,\max} = \frac{\Delta/L}{\left(1 + \frac{HL^2}{18I} \frac{G}{E}\right)} \quad (2)$$

where L , H , I , E and G are the length, height, second moment of area of the building cross-section, the Young's and the shear modulus of the equivalent beam, respectively, and t is the distance between the neutral axis and the edge

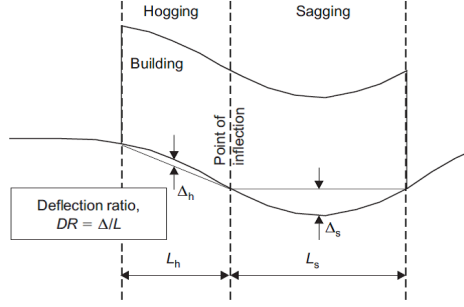


Figure 1: Deflection ratio for the sagging and hogging portions of the tunnelling-induced settlement trough [5].

of the beam in tension. Burland and Wroth [10] suggested an E/G value of 2.6 for masonry structures, assuming an isotropic elastic material with a Poisson's ratio of 0.3 for the equivalent beam. When applying the method to more flexible frame structures, they recommended a value of $E/G = 12.5$, although this method was later found to be unsuitable to capture the strain distribution [11]. Even for masonry structures, an effective E/G ratio is typically difficult to estimate, as it can be significantly affected by building dimensions and window openings, yet it strongly affects the calculation of the maximum bending and shear strains.

When predicting damage using the LTSM, differential vertical displacement is quantified by the deflection ratio Δ/L . As illustrated in Figure 1, Δ/L , and therefore the building strain, are calculated separately for the convex (sagging) and concave (hogging) portions of the settlement profile. Field data [12] has shown that buildings are generally more vulnerable to hogging than sagging deformations, mostly because in sagging the foundation offers an additional restraint to the settlement-induced deformation. For this reason, in Equations 1 and 2 the neutral axis is assumed to be in the middle of the beam in the sagging case ($t = H/2$) and at the lower beam edge in the hogging case ($t = H$) [10].

The beam horizontal strains are calculated as $\varepsilon_h = \delta/L$, where δ is the difference between the horizontal displacements of the greenfield profile at the two ends of the beam. The bending, diagonal and horizontal strains are then combined to obtain the total bending ε_{bt} and shear ε_{dt} strains:

$$\varepsilon_{bt} = \varepsilon_{b,\max} + \varepsilon_h \quad (3) \quad \varepsilon_{dt} = \frac{\varepsilon_h}{2} + \sqrt{\left(\frac{\varepsilon_h}{2}\right)^2 + \varepsilon_{d,\max}^2} \quad (4)$$

The larger of these two values is the total strain of the structure, which is compared to limit values to determine the damage class for the structure (Table 1).

2.1.2. Shear-based damage criterion

Son and Cording [4] developed a similar approach, focusing on the shear component of the building deformation. According to Boscardin and Cording [8], they measured the deformations of building units (generally delimited by transversal bearing walls for masonry structures or columns for concrete frames) in terms of the following deformation

Table 1: Damage classification system [7, 8, 13].

Category of damage	Damage class	Description of typical damage and ease of repair	Approximate crack width (mm)	Limiting tensile strain levels (%)
Aesthetic damage	Negligible	Hairline cracks of less than about 0.1 mm width.	up to 0.1 mm	0 - 0.05
	Very slight	Fine cracks which can easily be treated during normal decoration. Perhaps isolated slight fracturing in building. Cracks in external brickwork visible on close inspection.	up to 1 mm	0.05 – 0.075
	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repainting may be required externally to ensure water tightness. Doors and windows may stick slightly.	up to 5 mm	0.075 – 0.15
Functional damage, affecting serviceability	Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repainting of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired.	5 to 15 mm or a number of cracks > 3 mm	0.15 – 0.3
	Severe	Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floors sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15 to 25 mm, but also depends on number of cracks	> 0.3
Structural damage, affecting stability	Very severe	This requires a major repair involving partial or complete rebuilding. Beams loose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually > 25 mm, but depends on number of cracks	> 0.3

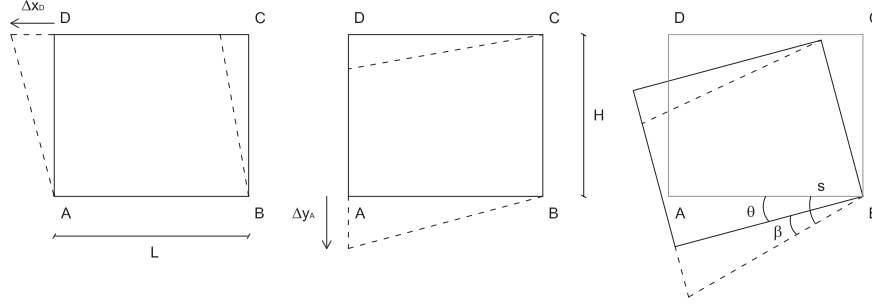


Figure 2: Deformation indicators (after Son and Cording [4]).

Table 2: Damage classification system [4].

Damage level	Damage	Critical tensile strain ε_c ($\times 10^{-3}$)
1	Negligible	0 – 0.5
2	Very slight	0.5 – 0.75
3	Slight	0.75 – 1.67
4	Moderate to severe	1.67 – 3.33
5	Severe to very severe	> 3.33

indicators (see Fig. 2): top horizontal strain $\varepsilon_{L,top} = \frac{\Delta x_D - \Delta x_C}{L}$, base horizontal strain $\varepsilon_{L,base} = \frac{\Delta x_A - \Delta x_B}{L}$, slope $s = \frac{\Delta y_A - \Delta y_B}{L}$, tilt $\theta = \frac{\Delta x_C - \Delta x_B}{H}$ and angular distortion $\beta = s - \theta$.

The maximum principal strain ε_p is calculated from the angular distortion β and the lateral strain ε_L : $\varepsilon_p = \varepsilon_L \cos(\theta_{max}^2) + \beta \sin(\theta_{max}) \cos(\theta_{max})$, where θ_{max} is the direction of crack formation and the angle of the plane in which ε_p acts, measured from the vertical plane: $\tan(2\theta_{max}) = \beta/\varepsilon_L$. The degree of damage is evaluated by comparing ε_p with defined values of critical tensile strain ε_c (Table 2).

2.2. Relative Stiffness Methods (RSMs)

Several researchers have proposed various procedures for extending LTSM methods to include soil-structure interaction by evaluating the relative stiffness of the building compared to the soil (Table 3). Potts and Addenbrooke [3] proposed a RSM that adjusts LTSM beam deflection using the following modification factors:

$$M^{DR,sag} = \frac{(\Delta_s/L_s)}{(\Delta_s/L_s)_{gr}} \quad M^{DR,hog} = \frac{(\Delta_h/L_h)}{(\Delta_h/L_h)_{gr}} \quad (5)$$

where Δ_s/L_s and Δ_h/L_h are the actual building deflection ratios, while $(\Delta_s/L_s)_{gr}$ and $(\Delta_h/L_h)_{gr}$ are the greenfield deflection ratios, i.e. the deflection ratios that would occur without the structure. Similarly, for the horizontal strains:

$$M^{\varepsilon_{hc}} = \frac{\varepsilon_{hc}}{\varepsilon_{hc}^{gr}} \quad M^{\varepsilon_{ht}} = \frac{\varepsilon_{ht}}{\varepsilon_{ht}^{gr}} \quad (6)$$

where ε_{hc} and ε_{ht} are the actual building horizontal strains in compression and tension, respectively, and ε_{hc}^{gr} and ε_{ht}^{gr} are the greenfield horizontal compressive and tensile strains. Potts and Addenbrooke [3] proposed design charts (Fig.

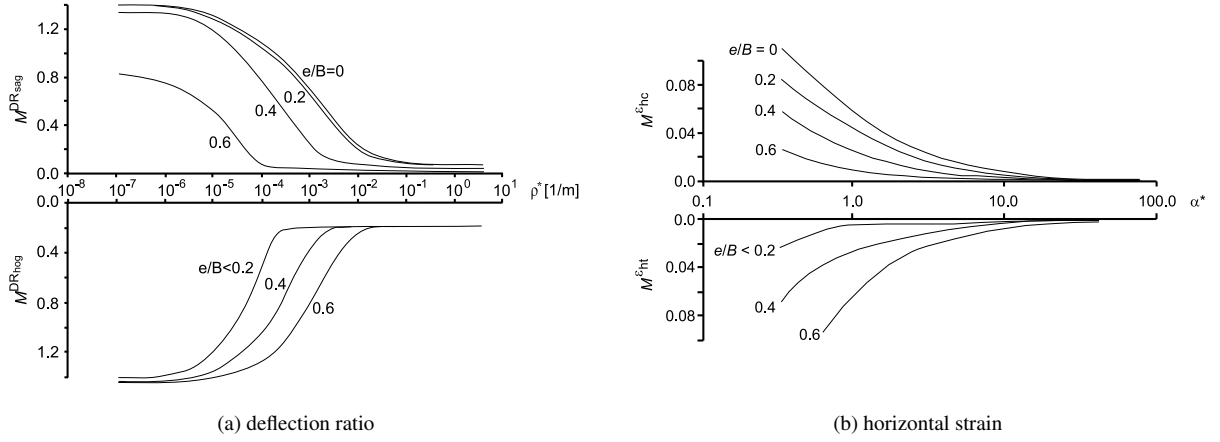


Figure 3: Design curves for modification factors [3]. Note that the dimension B corresponds to the building length L in the convention adopted in this paper.

3) to associate the modification factors to specific features of the building and the soil, summarised in the relative bending and axial stiffness:

$$\rho^* = \frac{EI}{E_s(L/2)^4} \quad (7) \quad \alpha^* = \frac{EA}{E_s(L/2)} \quad (8)$$

where A is the cross-section area, and E_s is the soil secant stiffness obtained at 0.01% axial strain in a triaxial compression test performed on a sample retrieved from a depth of $z_0/2$, where z_0 is the tunnel depth.

Franzius et al. [5] modified the relative stiffness, which was originally defined in plane strain conditions, to make it dimensionless in both two and three dimensions by including the effect of the tunnel depth and the building width B :

$$\rho_{\text{mod}}^* = \frac{EI}{E_s L^2 z_0 B} \quad (9)$$

$$\alpha_{\text{mod}}^* = \frac{EA}{E_s B L} \quad (10)$$

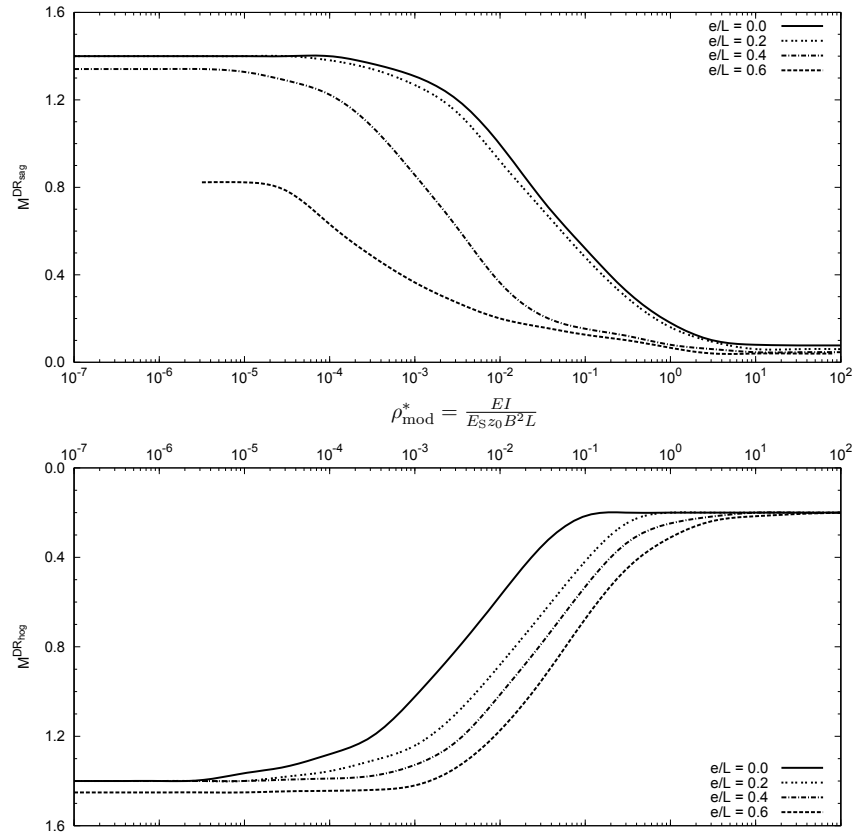
The design charts as modified by Franzius et al. [5] are shown in Figure 4.

Based on experimental tests and field data, Goh and Mair [6] partitioned the relative bending stiffness in the sagging and hogging zone of the greenfield settlement profile curvature (updated design charts are shown in Figure 5):

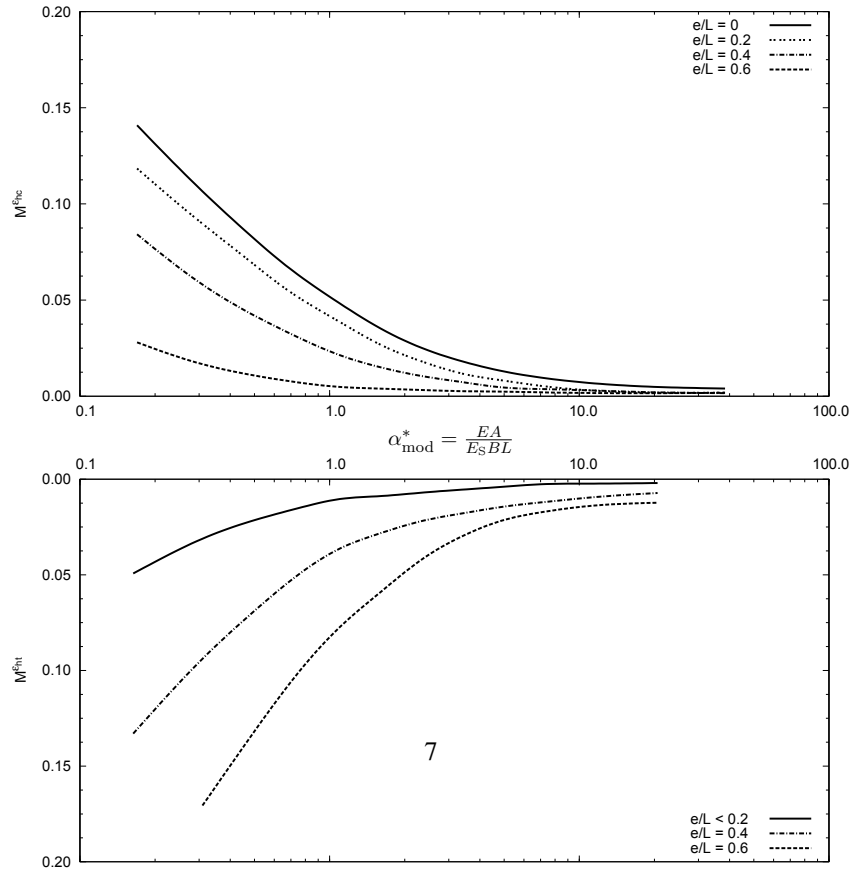
$$\rho_{\text{sag,par}}^* = \frac{EI}{E_s L_s^3 B} \quad \rho_{\text{hog,par}}^* = \frac{EI}{E_s L_h^3 B} \quad (11)$$

$$\alpha_{\text{par}}^* = \frac{EA}{E_s L} \quad (12)$$

Meanwhile, Son and Cording [4] took a notably different approach and proposed an RSM focused on the role of building shear stiffness in the soil-structure interaction; they developed an alternative definition of relative soil-building stiffness: $\frac{E_s L^2}{G H b_w}$, where E_s is the soil stiffness in the region of footing influence, G is the building elastic



(a) deflection ratio



(b) horizontal strain

Figure 4: Design curves for modification factors [5].

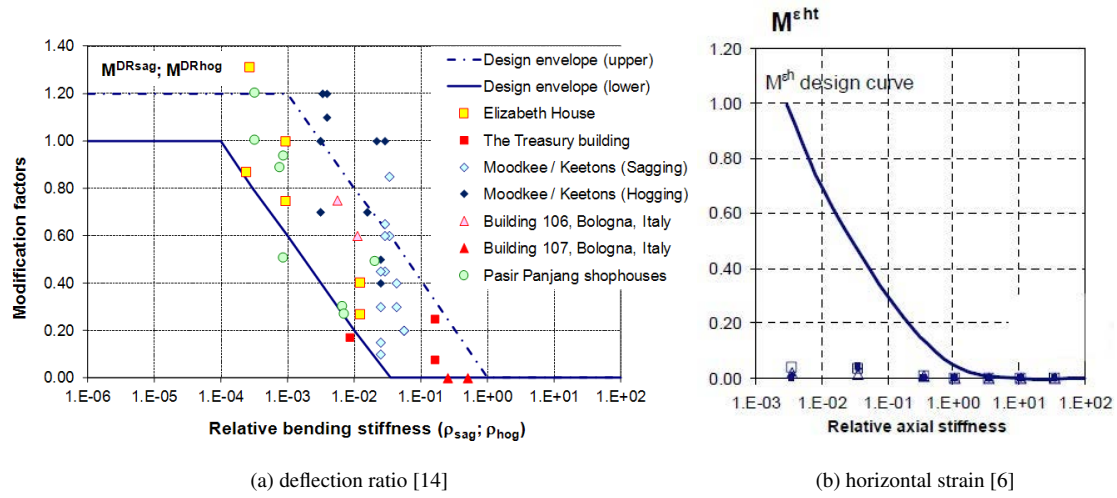


Figure 5: Design curves for modification factors.

shear modulus and b_w is the building wall thickness. Similarly to Potts and Addenbrooke [3], Son and Cording [4] developed a relationship between the relative stiffness and the building-induced reduction of greenfield deformation, though they quantified the deformation using the angular distortion β .

Table 3: Summary of main advantages and disadvantages of rapid methods for the assessment of settlement-induced damage to structures.

Method	Reference publication	Pros	Cons
Limiting Tensile Strain	Burland et al. [7], Boscardin and Cording [8], Burland et al. [13]	Rapid Easy to define geometrical input	Very conservative Difficult to define stiffness input No soil-structure interaction No opening effect
Relative stiffness	Potts and Addenbrooke [3], Son and Cording [4], Franzius et al. [5], Goh and Mair [6]	Soil-structure interaction	Difficult to define stiffness input

2.3. Calculation of building stiffness

One of the most difficult tasks in the application of the relative stiffness method is the determination of the overall bending stiffness of the building. An accurate calculation would require a detailed knowledge of the structural type, materials, construction techniques and current conditions; this information can be missing or not easily accessible at the time of the assessment. The task is made even more complicated by the need to select the most suitable calculation method among several proposed by different authors.

In their original formulation of the relative stiffness method, Potts and Addenbrooke [3] referred to two possible methods for the calculation of the building bending stiffness. For the method they predominantly used in their

analyses:

$$EI = E \sum_{i=1}^{n+1} (I_{\text{slab},i} + A_{\text{slab},i} d_i^2) \quad (13)$$

where i is the number of floor slabs, $I_{\text{slab},i}$ is the second moment of area of each slab, $A_{\text{slab},i}$ is the cross-sectional area of each slab, n is the number of storeys, d_i is the vertical distance between the structure's neutral axis and the slab's neutral axis. Since this formula can overestimate the stiffness of framed structures, they suggested the following alternative:

$$EI = E \sum_{i=1}^{n+1} (I_{\text{slab},i}) \quad (14)$$

where the shear transfer between the slabs is ignored, and the total building stiffness simply results from the sum of the bending stiffness of each slab. In both cases the axial stiffness EA is the sum of the axial stiffness values for each slab:

$$EA = E \sum_{i=1}^{n+1} (A_{\text{slab},i}) \quad (15)$$

The alternative approach suggested in Eq. 14 is based on the formula proposed by Meyerhof [15] for a multi-storey building frame: $EI = E \sum_{i=1}^n (I_i)$, where n is the number of storeys and I_i is the second moment of area of a single storey: $EI_i = EI_b \left[1 + \left(\frac{K_l + K_u}{K_b + K_l + K_u} \right) \frac{L^2}{l^2} \right]$. In the previous equation, $K_b = I_b/l_b$ is the average stiffness of beams, $K_l = I_l/h_l$ is the average stiffness of lower columns, $K_u = I_u/h_u$ is the average stiffness of upper columns, L is the building length, l_b is the beam length, $h = (h_l + h_u)/2$ is the average storey height.

Meyerhof [15] also included estimations for different building typologies. For multi-storey building frame with in-filled panels $EI = \sum_{i=1}^n \left[EI_i + \frac{(EI)_p L^2}{2h^2} \right]$, where $(EI)_p$ is the bending stiffness of the panel in the vertical plane, while for load bearing walls without openings where b_w is the average thickness of the wall and H is the height of the wall or the structure $EI = \frac{Eb_w H^3}{12}$.

Melis and Rodriguez Ortiz [16] developed an unified approach for different structural typologies:

$$EI = \sum (EI)_{\text{floors}} + \sum (EI)_{\text{walls}} + \sum (EI)_{\text{basements}} \quad (16)$$

where $(EI)_{\text{floors}} = E \left(\frac{1}{12} B b_s^3 + B b_s d^2 \right) \frac{1}{B}$ is the contribution of each floor slab, $(EI)_{\text{walls}} = E \left(\frac{1}{12} b_w H^3 + b_w H d^2 \right) \frac{1}{B}$ is the contribution of each wall, $(EI)_{\text{basements}} = E \left(\frac{1}{12} B b_b^3 + B b_b d^2 \right) \frac{1}{B}$ is the contribution of each foundation slab rigidly connected to the superstructure and $(EI)_{\text{basements}} = E \left(\frac{1}{12} B b_b^3 + B b_b c^2 \right) \frac{1}{B}$ is the contribution of each foundation slab in case of basement hinged to the superstructure.

In these equations, b_s and b_b are the slab and basement thickness, respectively, d is the distance from the slab to the neutral axis of the structure, assumed at the mid-height of the structure, and c is the distance from the assumed hinge between the superstructure and the basement. The second moment of area of walls, continuous footings and/or foundation slabs forming the basement are calculated relative to the neutral axis of the basement. The second moment of area of each slab relative to its own middle plane is typically neglected, as well as the stiffness of columns in

Table 4: Reduction factor on building bending stiffness EI due to openings [16].

Type of wall	Length < $2H$	Length > $2H$
No openings	1.00	1.00
Openings from 0 to 15%	0.70	0.90
Openings from 15 to 25%	0.40	0.60
Openings from 25 to 40%	0.10	0.15

conventional frame structures. Partition walls are also neglected, due to their reduced stiffness; in case of internal bearing walls, they can be included as external walls. A novelty in the approach by Melis and Rodriguez Ortiz [16] was the inclusion of reduction factors to consider the effect of door and window openings. The reduction factors are dependent on the percentage of openings and the aspect ratio of the structure (Table 4).

In the numerical study that led to the formulation of Equations 9 and 10, Franzius et al. [5] considered only concrete frames. By assuming the neutral axis at the mid-length of the building they calculated the bending and axial stiffness according to Eq. 13 and 15, respectively. Mair and Taylor [17] evaluated the building stiffness of historic buildings adjacent to the Jubilee Line Extension as $EI = E \frac{bH^3}{12}$, where b is the unit building width.

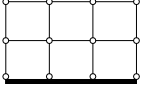
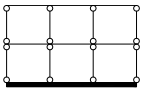
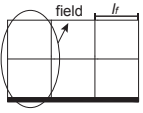
Dimmock and Mair [18] later modified the calculation of the bending stiffness for masonry structures on shallow strip foundations in the hogging zone by neglecting the wall contribution: $EI = E \frac{bh_f^3}{12}$, where h_f is the height of the foundation, and not the entire height of the masonry wall. Furthermore, in the sagging zone, they proposed a 90% reduction in the masonry wall stiffness.

The equivalent tensile strains calculated with the LTSM (Eqs. 1 and 2) for massive walls are conceptually inconsistent if applied to frame structures. Furthermore, the effect of the modified E/G factor on the tensile strain depends on the L/H ratio of the building and not representative for all kinds of frame structures. Therefore, Netzel [11] presented a new approach to evaluate the influence of the imposed settlements on the beams and columns of a frame structure. The maximum bending moment M_{\max} and shear force V_{\max} of the fictitious beam are calculated as:

$$M_{\max} = \frac{\Delta}{L} \frac{12EI}{L} \quad V_{\max} = \frac{\Delta}{L} \frac{24EI}{L^2} \quad (17)$$

For the calculation of the equivalent second moment of area, three types of frame structures are considered, depending on the structural connections between floor and columns (Table 5). The maximum value of bending moment and shear force calculated with Equations 17 are redistributed to the structural elements, considering the different distribution of stiffness in the three frame typologies (Table 5). The structure is then verified for the increased values of bending moments and shear forces.

Table 5: Calculation of equivalent second moment of area and additional bending moment and shear force for frame structures [11].

Frame type	I equivalent	Additional M and V
Hinged connections between beams and columns	$I = I_b$	$M_b = \frac{I_b}{I} M_{\max} \quad V_b = \frac{A_b}{A} V_{\max}$
 <p>where I_b is the second moment of area of the continuous foundation plate</p>		<p>where M_b and V_b are the additional moment and shear force concentrated in the foundation plate, respectively, and A_b is the cross-sectional area of the foundation plate</p>
Hinged connections between columns or walls and continuous beams	$I = I_b + \sum_{i=1}^n I_{s,i}$	$M_b = \frac{I_b}{I} M_{\max} \quad V_b = \frac{A_b}{A} V_{\max}$ $M_{s,i} = \frac{I_{s,i}}{I} M_{\max} \quad V_{s,i} = \frac{A_{s,i}}{A} V_{\max}$
 <p>where I_s is the second moment of area of one floor slab</p>		<p>where $M_{s,i}$ and $V_{s,i}$ are the additional moment and shear force concentrated in each floor slab, respectively, and $A_{s,i}$ is the cross-sectional area of one floor slab</p>
Full monolith connections between beams and columns	$I = I_b + \sum_{i=1}^n \left[I_{s,i} \left(1 + \frac{1}{\frac{I_{s,i}/l_{f,i}}{I_{u,i}/h_{u,i} + I_{l,i}/h_{l,i}} + 1} n_f^2 \right) \right]$	A numerical analysis is required to evaluate the redistribution of the moment and the shear forces to the individual structural elements
 <p>where n_f is the amount of fields, $l_{f,i}$ is the length of one field, $I_{u,i}$ and $I_{l,i}$ are the second moment of area of column/wall above and below considered floor slab, and $h_{u,i}$ and $h_{l,i}$ are the storey height above and below the considered floor slab, respectively</p>		

3. Sensitivity study

To quantify the effects of the assumptions described in the previous section, this section presents a parametric analysis performed on a number of building models with different structural features. The LTSM [7] and three RSMs [3, 5, 6] were applied.

3.1. Tunnelling-induced settlements

Tunnelling-induced greenfield settlements were calculated according to Peck [19]:

$$S_v(x) = S_{v,\max} e^{-\frac{x^2}{2i^2}} \quad (18)$$

where $S_{v,\max}$ is the maximum settlement measured above the tunnel axis, x is the horizontal distance from the tunnel axis and i is the horizontal distance between the tunnel axis and the point of inflection of the settlement trough. By defining the volume loss percentage V_L as a function of the volume of ground lost V_S per meter of tunnel and the diameter D : $V_L = \frac{V_S}{\pi D^2 \cdot \frac{4}{z_0}}$. $S_{v,\max}$ can be derived as $V_L = \frac{0.313 V_S D^2}{i}$, where $i = k z_0$, k is the trough width parameter and z_0 is the tunnel depth. The horizontal component S_h of the ground displacement were calculated according to O'Reilly and New [20]:

$$S_h(x) = -\frac{x S_v(x)}{z_0} \quad (19)$$

The 1 mm cut-off and the splitting at the inflection point of the settlement trough, as proposed by Mair et al. [2], were applied.

3.2. Damage classification

The deflection ratio and average horizontal strain were used to calculate the limiting tensile strain for each structure (Section 2.1). For all structures it was assumed that $E/G = 2.6$. The bending stiffness of each structure was calculated according to Melis and Rodriguez Ortiz [16] (Eq. 16) by assuming that the global neutral axis was at the mid-height of the structure. The same reduction depending on façade openings was applied to the axial stiffness. Equations 7,9 and 11 were used to calculate the relative stiffness for the three considered RSMs; the modification factors were then derived from the corresponding design charts (Figs. 3, 4, 5). Based on these modification factors and on the greenfield deformations (Section 3.1), the actual deflection ratio and horizontal strain of the building were calculated and used to determine the total tensile strain.

3.3. Reference structure

The geometry of the reference case is illustrated in Figure 6. The structural features are based on a typical masonry Georgian town house with strip foundations. The wooden floor beams were assumed to run perpendicularly to the building direction and therefore to have negligible impact on the global stiffness of the structure. The tunnel has

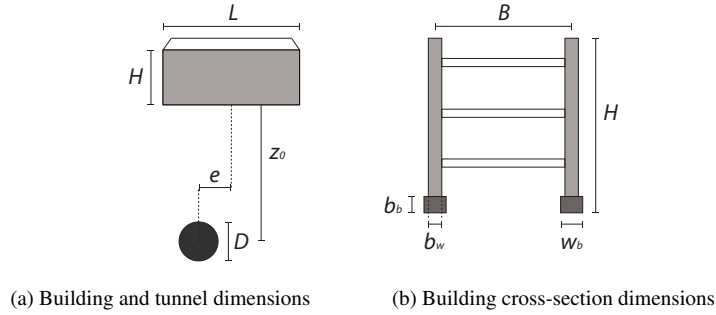


Figure 6: Reference structure geometry.

depth and diameter similar to the Crossrail tunnels and the soil parameters are the same as assumed by Potts and Addenbrooke [3]. Table 6 reports the dimensions and parameters of the reference model. Figure 7 compares the relative stiffness and modification factor values for the reference buildings calculated accordingly to the analysed RSMs. The envelope defined by Goh and Mair [6] is shown for reference.

Table 6: Sensitivity study: reference model parameters.

Component	Variable	Value
Building	Length L	25 m
	Height H	10 m
	Width B	10 m
Façade	Thickness b_w	0.25 m
	Stiffness E_w	3 GPa
	Openings ratio O	20%
Foundation	Thickness b_b	0.3 m
	Width w_b	0.4 m
	Stiffness E_b	3 GPa
Tunnel	Depth z_0	25 m
	Diameter D	7.18 m
	Volume loss V_L	1%
Soil	Trough width parameter k	0.5
	Reference stiffness E_s	124.5 MPa

3.4. Variable parameters

The sensitivity study aimed to investigate the effect of the following parameters on the field damage assessment:

- **Building height** By keeping constant the reference length L of the building and varying the building height H , the influence of the L/H ratio and therefore the bending-shear strain decomposition on the damage assessment was evaluated. The selected range includes building from one to six storeys.

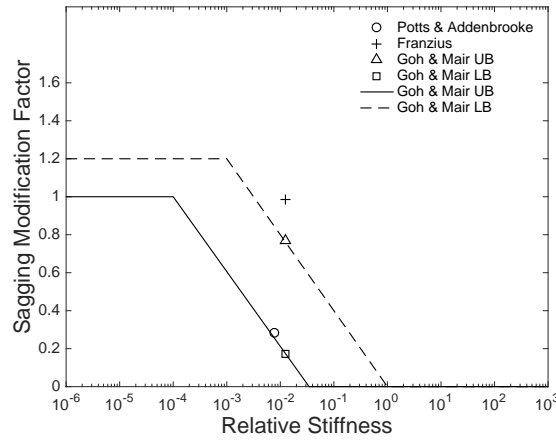


Figure 7: Relative stiffness and modification factor for the reference building: comparison between RSMs. Note that Potts & Addenbrooke's relative stiffness is given in [1/m], while the Franzius' and Goh & Mair's values are dimensionless.

- **Building eccentricity** Changing the building position with respect to the tunnel axis enabled assessment of the effect of splitting the settlement profile into the sagging and hogging parts for the different damage assessment methods.
- **Opening ratio** The influence of the building openings was quantified by varying the ratio between window openings and total area of the façade. The selected range of variations spans from façades with no openings to façades with half of their surface covered by windows.
- **Tunnel depth** The tunnel depth affects both the horizontal displacement trough Equation 19 and the vertical displacement trough Equation 18, via the location of the point of inflection $i = kz_0$. The selected range of tunnel depth corresponds to a minimum cover-to-diameter (C/D) ratio of 1.6 (shallow tunnel) to a maximum of 5 (deep tunnel).

Furthermore, the impact of potential errors in the estimation of the material parameters was taken into account by varying the **soil stiffness**, **masonry stiffness** and **trough width parameter**. The variation range includes typical values for historic masonry. The soil stiffness range includes typical values from granular material to clay. Similarly, a large variety of soils from sand to clay have been considered for the trough width parameter. The ranges for all assumed variations are listed in Table 7. Each parameter variation was performed for three different values of building length (15, 25 and 35 m) and three volume losses (0.1, 0.15 and 0.2 %). A total number of 7128 configurations was analysed. Each parameter variation was performed for three different values of tunnel depth (15, 25 and 40 m), building height (5, 10 and 20 m), building length (15, 25 and 35 m), opening ratio (0, 20 and 50%) and volume loss (0.1, 0.15 and 0.2 %). A total number of 192456 configurations were analysed. Sections 3.6.1 to 3.6.5 discuss selected results which illustrate the most significant trends. When not specified as varying, the presented parameters correspond to the reference values (Table 6).

Table 7: Sensitivity study: parametric ranges.

Variable	Lower limit	Upper limit
Building height H	5 m	20 m
Building length L	15 m	35 m
Building eccentricity e	-24.5 m	37.5 m
Openings ratio O	0%	50%
Volume loss V_L	1 %	2 %
Tunnel depth z_0	15 m	40 m
Masonry stiffness $E_m = E_w = E_b$	1 GPa	6 GPa
Soil ref. stiffness E_s	25 MPa	175 MPa
Trough width parameter k	0.2	0.7

3.5. Global sensitivity study

To quantify the overall sensitivity of the damage assessment outputs to the tunnel and building geometrical parameters, and to the uncertainties in material parameters, a total effect sensitivity index [21] was calculated for each of the factors listed in Table 7. The use of a total sensitivity index allowed to evaluate the model sensitivity over the entire input parameter space and quantify the effect of each input parameter and its coupling with other input parameters.

Similarly to recent applications of global sensitivity analyses to geotechnical problems [22], the calculation of the total effect sensitivity index S_{Ti} was based on two independent (N, P) matrices \mathbf{A} and \mathbf{B} , each one containing N random samples of the input parameter vector $\mathbf{X} = X_1, X_2, \dots, X_P$, and P (N, P) matrices \mathbf{C}_i , each one equal to the matrix \mathbf{B} but with its i th column copied from \mathbf{A} . For each of the P parameters i , S_{Ti} was calculated as [22]:

$$S_{Ti} = \frac{(\mathbf{y}_B - \mathbf{y}_{Ci})^T (\mathbf{y}_B - \mathbf{y}_{Ci})}{2\mathbf{y}_B^T \mathbf{y}_B - 2N(\bar{\mathbf{y}}_B)^2} \quad (20)$$

where \mathbf{y}_B and \mathbf{y}_{Ci} are vectors containing the model evaluation for matrix \mathbf{B} and \mathbf{C}_i , respectively, while $\bar{\mathbf{y}}_B$ is the mean of the values contained in \mathbf{y}_B .

The (N, P) matrices \mathbf{A} and \mathbf{B} were randomly generated by using a uniform probability density function, within the ranges reported in Table 7, for $N=10,000$ samples and $P=9$ parameters. The corresponding sagging and hogging modification factors and the total strain values were used as model evaluation.

3.6. Results

The outcomes of the parametric study were compared in terms of influence of the considered parameters.

3.6.1. Building stiffness

Figures 8 and 9 show the effect of opening percentage, tunnel cover-to-diameter (C/D) ratio and building height-to-length (H/L) on the building bending and axial stiffness calculation. An increase in opening percentage has a significant influence in reducing the stiffness; the sharp drop in stiffness at 40% openings is due to the façade contribution being completely neglected when the openings are more than 40% of the total façade area [16]. The facade

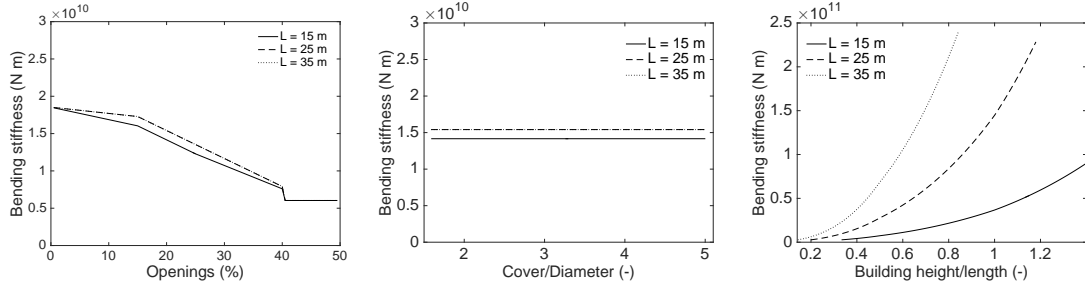


Figure 8: Sensitivity study: bending stiffness variation with openings, C/D and H/L ratio.

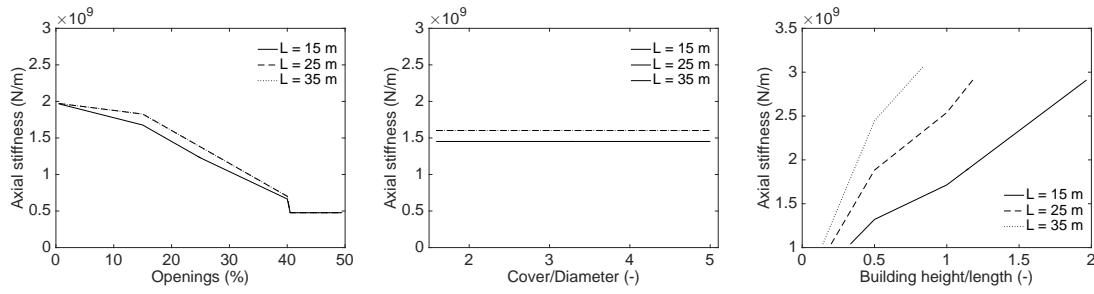


Figure 9: Sensitivity study: axial stiffness variation with openings, C/D and H/L ratio.

stiffness is proportional to H^3 , so it is expected that height has the largest effect on the building stiffness. The building axial stiffness is roughly proportional to H .

The tunnel depth does not play any role in the stiffness calculation. However, the plots including varying C/D ratio clearly show that the axial and bending stiffness are slightly dependent on L . Although L is not a parameter directly included in the stiffness formulation, the reduction factors [16] are dependent on the H/L ratio, as well as on the percentage of openings.

3.6.2. Building located symmetrically above tunnel (sagging case)

For a building located symmetrically above the tunnel (i.e., $e = 0$), three different building lengths were again considered and the parameters were varied. Note that for the variations in opening percentage and building H/L ratio, the inflection point of the greenfield settlement occurs at 12.5 m from the tunnel centerline, so the building is in sagging. For the variation of tunnel C/D ratio, the position of the inflection point varies from 7.5 to 40 m.

Figure 10 shows significant variability in the sagging modification factors predicted by the RSMs considered. These variations are due to the differing modification factor charts, but also due to the different methods of calculating the relative stiffness (Eqs. 7, 9, 11). In general, Franzius et al. [5] gave the most conservative assessment, while Potts and Addenbrooke [3] usually predicted a modification factor between the upper and lower bound estimations of Goh

and Mair [6]. All methods were affected by the building length, as expected based on the relative bending stiffness formulations.

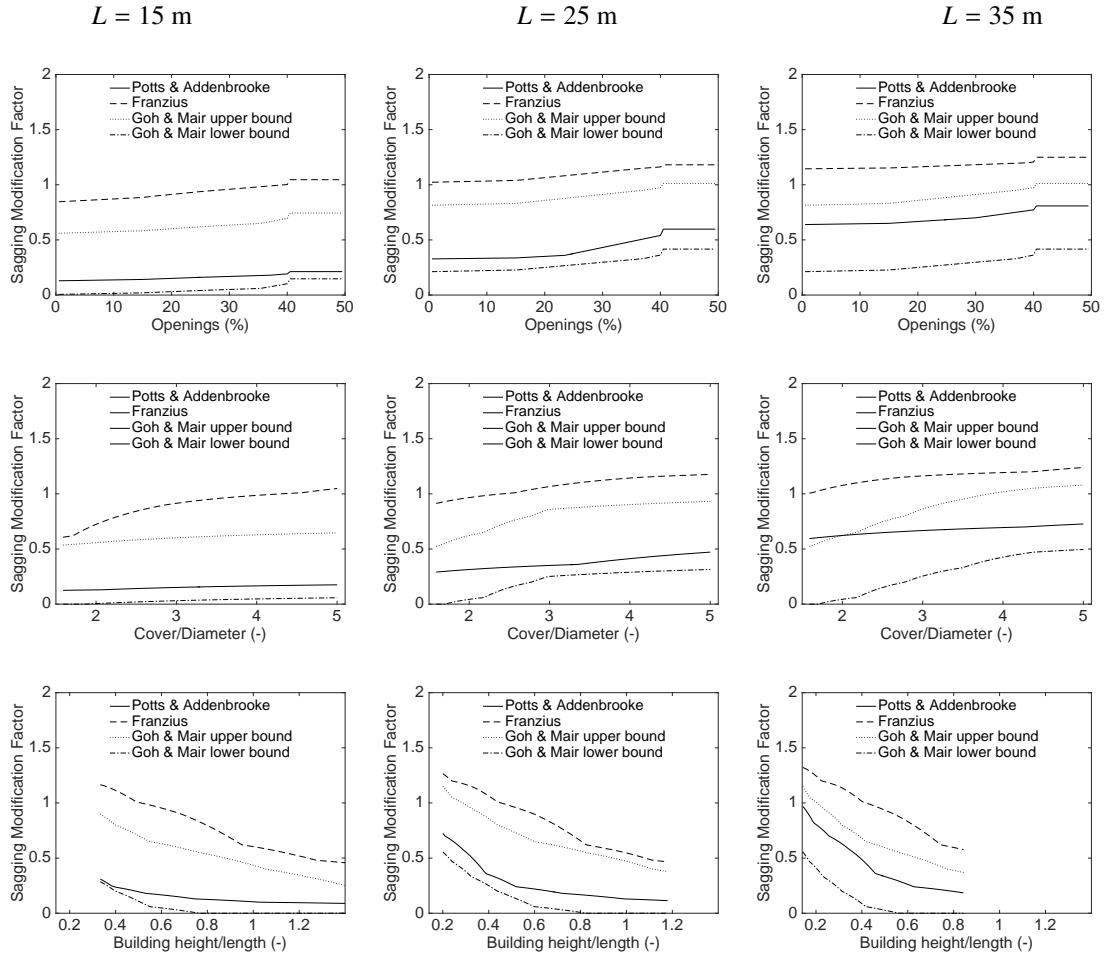


Figure 10: Sensitivity study results, sagging: variation of deflection ratio modification factors with openings, C/D and H/L ratio.

The trends in response to varying parameters were similar, although the sensitivity of the RSMs varied. For example, the Potts and Addenbrooke [3] formulation is less sensitive to tunnel depth than the other methods. The tunnel depth affects the relative stiffness both directly and indirectly. In all RSM formulations, the soil stiffness increases with tunnel depth, so the relative stiffness (Eqs. 7 to 12) decreases. This causes the building to deform more closely to the ground movements, increasing the modification factor. This trend is similar for all RSMs. However, in addition to this effect, Franzius et al. [5] directly included the tunnel depth in the relative bending stiffness (Eq. 9), and Goh & Mair's partitioning method causes a change in L_s with tunnel depth, causing a more sensitive response to tunnel depth. Numerous other effects of the RSM assumptions can be similarly evaluated using Figure 10.

The predicted horizontal strain modification factors are essentially zero for the range of parameters considered in Figure 10 (plots not shown for brevity). Thus, the horizontal strain is negligible for almost all cases considered. The only exception is the case where the stiffness of the façade is completely neglected because of an opening ratio $> 40\%$, so the only contribution to the axial stiffness is given by the shallow foundation.

Figure 11 shows the predicted total tensile strain for 1% surface volume loss for the same variation of parameters as in Figure 10. The total strains labelled greenfield were derived from the greenfield displacements and are dependent on the tunnel properties and the external building geometries, as defined in Equations 1 to 4, but are independent of the building stiffness and therefore the opening percentage. Only 1% volume loss results are included because that was the typical volume loss used for Crossrail building damage predictions. Additional volume losses were also investigated, but mostly show similar trends to Figure 11, with an increase in total tensile strain for higher volume losses. In Figure 11, the significant reduction of strain predicted by all the RSMs is dominated by the fact that the horizontal strain modification factor is approximately zero for nearly all cases considered. For many cases, the deflection ratio modification factors alone would not predict such a drastic reduction.

For the sagging case, the variation of strain with the building height depends on the building length. This is due to the combined effect of the H/L and Δ/L ratios on the bending and shear components of the strain (Eqs. 1 and 2). This effect can be observed in the variation of the greenfield curve, which is decreasing with the building H/L ratio for relatively small H/L values and increasing with H/L for larger H/L values. In the RSMs, the reduction in strain with H/L is amplified by the dependence of the building stiffness on H^3 .

The variation of tunnel C/D ratio has the most significant impact on the total tensile strain. The tunnel depth affects the relative stiffness both directly and indirectly, as indicated above. Furthermore, it influences the calculation of the deflection ratio. A reduction in z_0 reduces the spacing of the inflection point $i = kz_0$; this results in an increased curvature of the settlement profile, which is quantified by the increase in Δ/L . Depending on the building length, a reduction in z_0 from 35 to 15 m can lead to an increase of the LTSM predicted strain larger than 100%. This factor is particularly relevant near to stations, where it is convenient to construct the tunnels as close as possible to the surface, to minimise the depth of station boxes and reduce the costs. In these areas, tunnels are shallower and construction techniques generally result in greater volume losses; therefore, the LTSM is expected to predict a relatively high level of potentially vulnerable buildings. When the RSMs are applied, the predicted strain decreases between 40% and 90%, depending on the specific method and the building length.

3.6.3. Building located in pure hogging region

The same building model and variable ranges were used to analyse the pure hogging case. The eccentricity of the structure is defined as $e = kz_0 + \frac{L}{2}$. Figure 12 shows the influence of opening ratio, tunnel C/D ratio and building H/L ratio on the deflection ratio and horizontal strain modification factors, respectively. In this case, e is constant for the variations of opening and H/L ratio, but since k is defined by the tunnel depth, e varies with C/D so that the structure

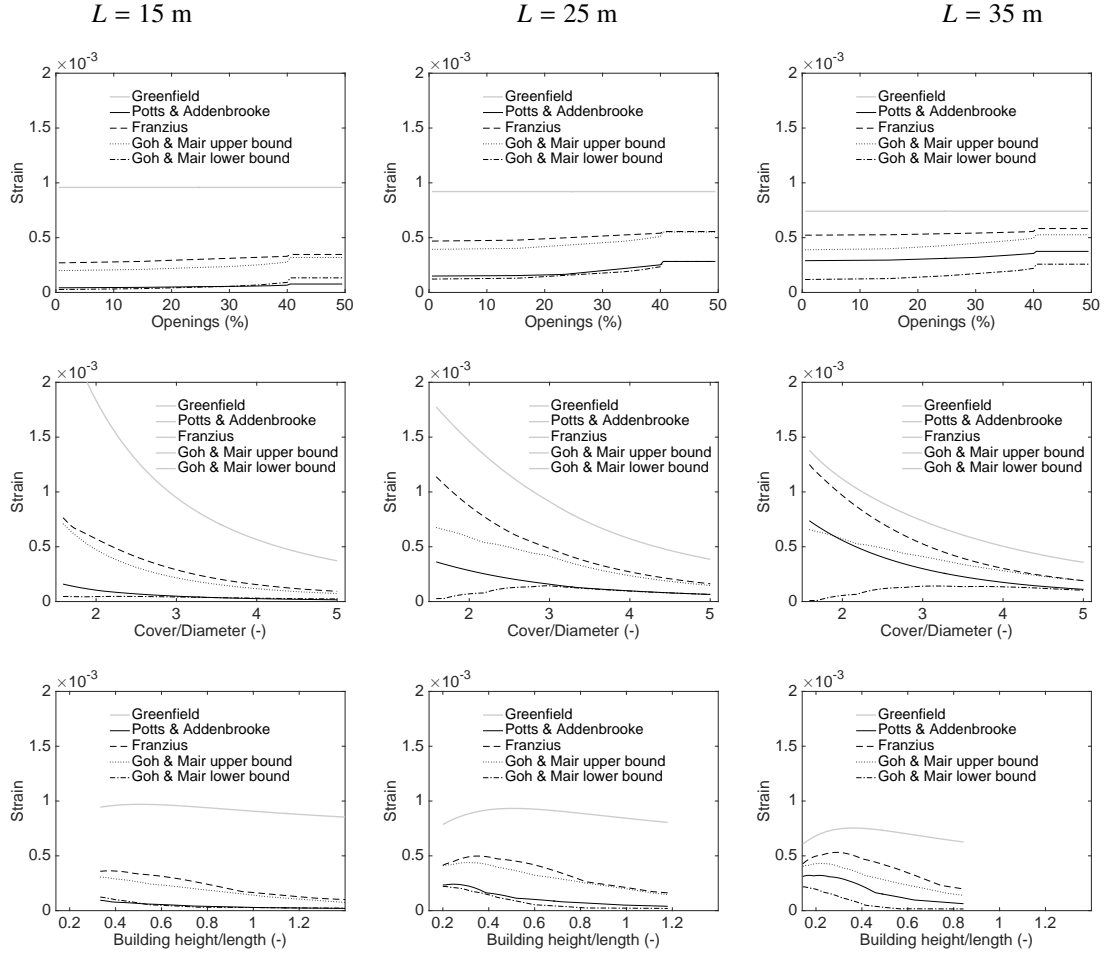


Figure 11: Sensitivity study results, sagging: total strain variation with openings, C/D and H/L ratio, $V_L = 1\%$

is always subjected to the pure hogging profile which exhibits the maximum differential settlement along the building length.

An increased relative eccentricity e/L needs to be considered when using the design charts by Potts and Addenbrooke [3] and Franzius et al. [5] (Figs. 3a and 4b). For each design chart, a 2D interpolation of the modification factor matrices was performed by using the relative stiffness and the relative eccentricity as reference values. The increased e/L ratio leads to higher modification factors for the corresponding variations of the relative stiffness method, even if the relative bending stiffness is the same as in the sagging case. Goh and Mair [6] use the same design charts but define different relative stiffness for the hogging and sagging case (Eq. 11).

Since L_h is defined by the location of the inflection point and the 1 mm cut off point (approximately at $2.5i$, where $i = kz_0$), a reduction in tunnel depth z_0 leads to a reduction in L_h . The effect of decreasing the length of the hogging zone is evident when C/D is smaller than 2.8.

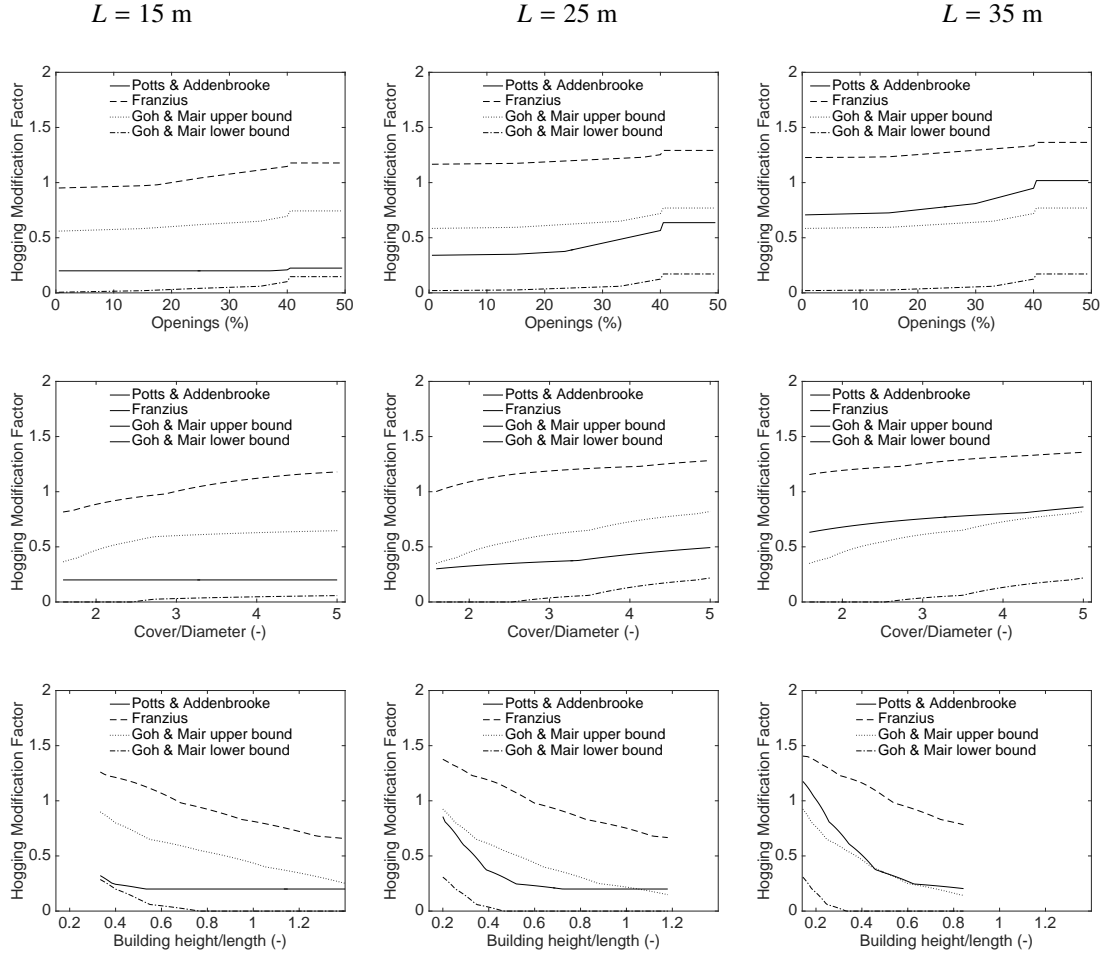


Figure 12: Sensitivity study results, hogging: variation of deflection ratio modification factors with openings, C/D and H/L ratio.

The hogging modification factors reported in Figure 12 show trends similar to the sagging modification factors (Fig. 10). Additionally, the Franzius RSM predicted the largest modification factor, while the Potts & Addenbrooke's assessment again tended to fall between the Goh & Mair upper and lower bound curves. However, for longer buildings ($L=35$ m), the different weight of L_h in Goh & Mair's relative stiffness formulation (Eq. 11), combined with Goh & Mair's design charts (Fig. 5), caused the Goh & Mair upper and lower bounds to give the lowest hogging modification factors. The predicted horizontal strain modification factors are again essentially zero, apart from when opening percentages are greater than 40%.

The differences observed between sagging and hogging modification factors propagate to the strain calculation. Figure 13 shows the strain prediction for a structure in pure hogging at 1% volume loss. Due to the reduced deflection ratio of the greenfield settlement trough in the hogging zone, the predicted strain is lower in hogging than in sagging. The general variations with openings, C/D and H/L ratio are similar to the ones observed in the sagging case (Fig. 11), and the significant reduction in strain compared to the greenfield is again dominated by the horizontal modification factor being approximately zero.

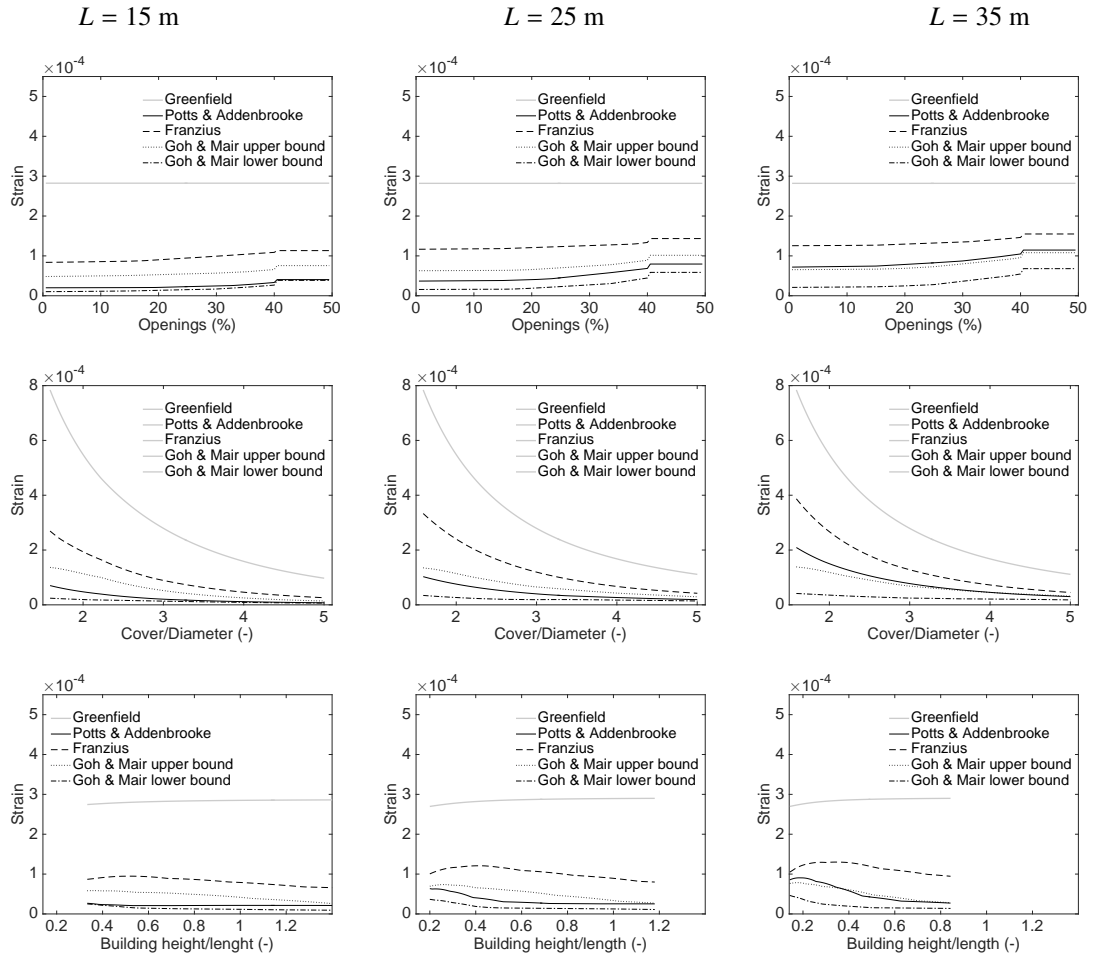


Figure 13: Sensitivity study results, hogging: total strain variation with openings, C/D and H/L ratio, $V_L = 1\%$.

3.6.4. Eccentricity

Sections 3.6.2 and 3.6.3 discuss buildings located primarily in hogging or sagging, while this section considers a smooth variation of building position. Potts and Addenbrooke [3] and Franzius et al. [5] design curves vary with the relative eccentricity e/L ; higher relative eccentricities result in smaller modification factors for the sagging case and larger modification factors for the hogging case. Goh and Mair [6] design curves are independent from the building

eccentricity. In their formulation, e affects the final assessment through its influence on the sagging and hogging building length: shorter L_s and L_h lead to smaller $\rho_{\text{sag,par}}^*$ and $\rho_{\text{hog,par}}^*$.

Figure 14 shows the variation of the modification factors and the total tensile strain as a function of the building eccentricity over the trough width (e/i) ratio for increasing values of building length. A volume loss of 1% is considered. Potts and Addenbrooke [3] and Franzius et al. [5] modification factors directly reflect the different design curve trends for the hogging and sagging case: sagging modification factors decrease and hogging modification factors increase with an increase in eccentricity. Similarly, Goh and Mair [6] modification factors directly depend on the sagging and hogging relative stiffness definitions (Eq. 11).

For buildings centered near the inflection point, where the curvature of the settlement trough is minimum, the strain decreases significantly for both the LTSM and RSM methods. Since the average curvature over a longer building is always less than the maximum curvature at the tunnel axis, increasing the building length in the LTSM framework results in a lower greenfield deflection ratio and therefore a lower maximum strain that occurs over a wider range of eccentricities. Total strains are progressively larger for increasing volume losses, but the trends are similar (plots not shown).

3.6.5. Uncertain material parameters

Several material parameters which are involved in the relative stiffness calculation are difficult to assess without a significant amount of testing. Figure 15 quantify the effect of potential uncertainties in the trough width parameter k , soil stiffness E_s and masonry stiffness E_m on the damage assessment. A 25 m long building located symmetrically above the tunnel and a volume loss of 1% are considered.

The soil properties (k and E_s) have no effect on the bending and axial building stiffness. Assuming that both the façade and foundation are masonry, the building bending and axial stiffness are proportional to the masonry stiffness. The trough width parameter has no effect on the sagging modification factors for Potts and Addenbrooke [3] and Franzius et al. [5], while it significantly affects the prediction by Goh and Mair [6]. Varying k influences the location of the inflection point and therefore changes the maximum size of the sagging zone. In the presented case, when $k > 0.5$ the structure is fully contained within the sagging zone of the greenfield settlement trough and therefore the modification factor remains constant.

All the relative stiffness formulations are inversely proportional to the soil stiffness E_s . The considered range is much larger than the typical variation for an individual project, as the extreme values here refer to soft Singapore clay and very stiff London clay. Assuming a more realistic range of 50 MPa for an individual tunnelling project, the modification factors can vary by 15%. All the modification factors decrease with the increase in masonry stiffness, as expected. The horizontal modification factors are again essentially zero across the entire range of all parameters considered (plot not shown), allowing the horizontal strain to be neglected, again provided that openings are less than 40%.

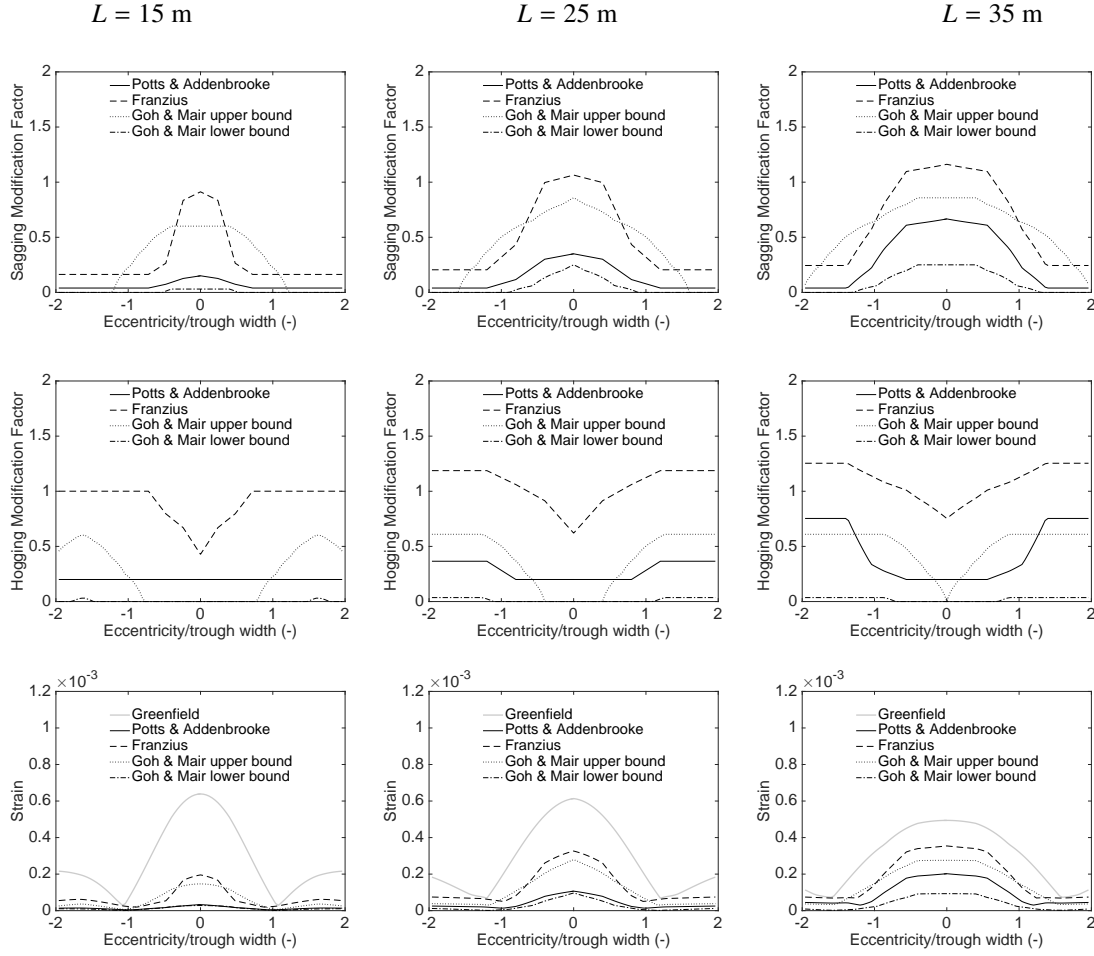


Figure 14: Sensitivity study results, total strain vs e/i ratio.

Assuming a constant E/G ratio of 2.6, the considered variation of soil and masonry stiffness has a relatively small effect on the LTSM total strain prediction. The total strain is again significantly smaller than the greenfield predictions because the predicted horizontal strain is essentially zero due to its very low modification factor, combined with the sagging modification factor. In general, the total strain predictions are most sensitive to uncertainties in the trough width parameter. This is because changing the trough width significantly affects the LTSM strain, which subsequently affects all RSM predictions. Consistently with previous observations, the structure exhibits increasing tensile strains for increasing values of volume loss (plots not shown).

3.6.6. Total effective sensitivity index

As described in Section 3.5, the sensitivity of the model to the analysed parameters (Table 7) was quantified by the total effective sensitivity index S_{Ti} (Figures 16 and 17). Since $\sum_{i=1}^P S_{Ti} \geq 1$, the indices were normalised as

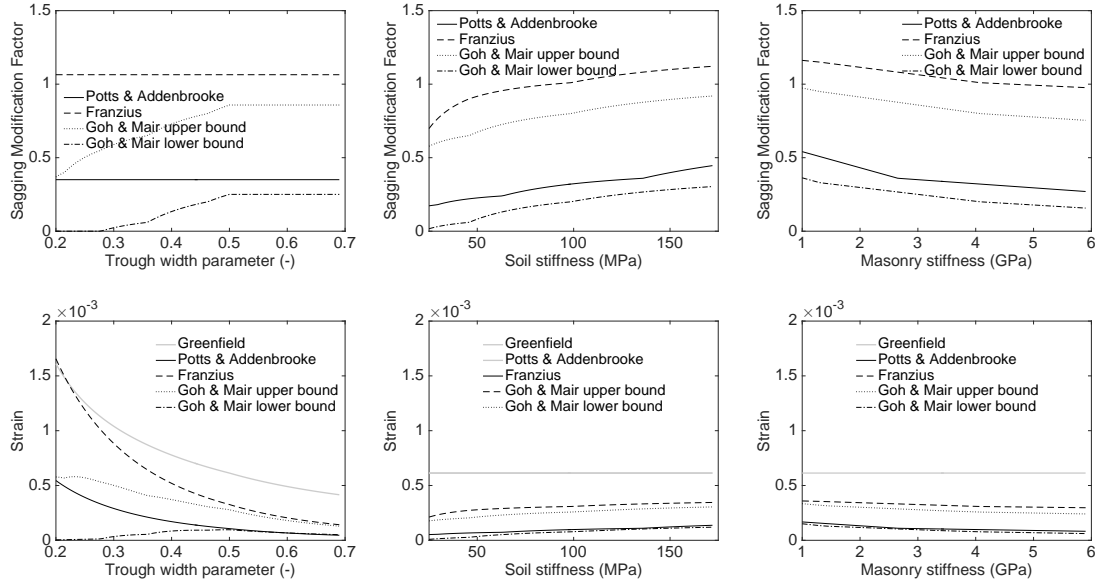


Figure 15: Sensitivity study results, modification factors vs uncertain material parameters.

331 $S_{Ti,n} = \frac{S_{Ti}}{\sum_{i=1}^P S_{Ti}}$. The relationship between input and output variations was evaluated both in terms of modification
 332 factors and strain values for all the considered RSMs.

333 For the tunnel and building parameters (Figure 16), the global sensitivity analysis was used to generalise the
 334 observed trends to the entire space of input variations, removing the potential dependency from the reference building
 335 parameters. Figures 16a and 16b confirm the different variability of the different RSMs, and Figure 16c validates the
 336 appropriateness of the governing factor selection. As expected, the modification factors exhibit a high sensitivity to
 337 the structural parameters (e.g. the building height, since the building stiffness depends on H^3), while the variation

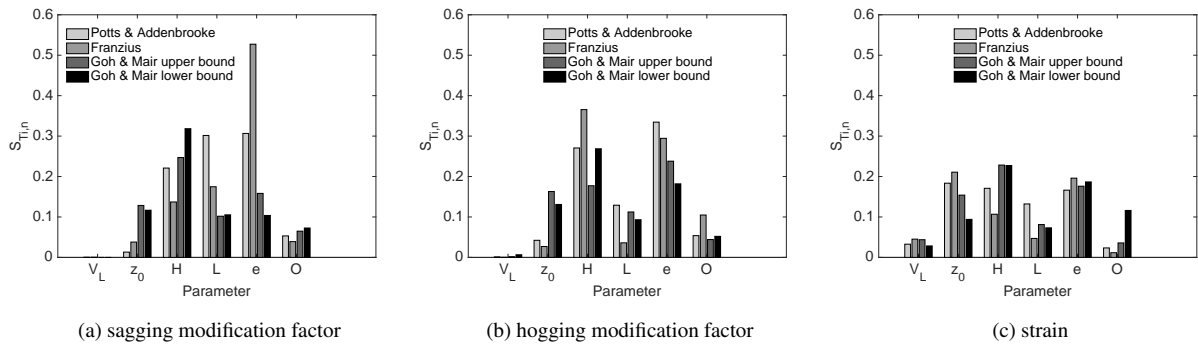


Figure 16: Total effective sensitivity index for tunnel and building geometrical parameters.

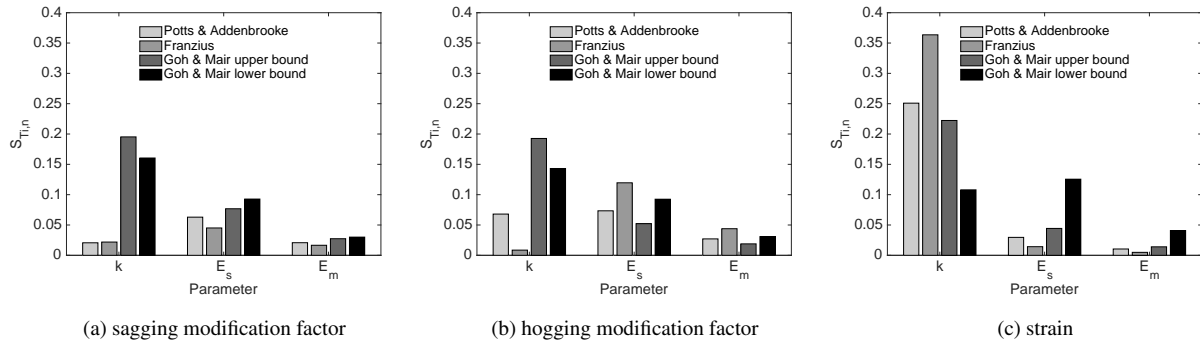


Figure 17: Total effective sensitivity index for material parameters.

of volume loss has a negligible effect, since it impacts the greenfield and structure-affected settlement profile in a proportional way. The strains are subjected to a generally more equal dependency on the input parameters, e.g. confirming the relatively high impact of the tunnel depth.

Differently from the geometrical parameters, the estimation of building and soil material parameters can be affected by a relatively high level of uncertainties. The global analysis was therefore used in this case to quantify the effect of parameter uncertainties on the output variations. Figure 17 confirms the previously observed trends with respect to the modification factors and strain values, e.g. highlighting the generally larger effect of k on the final strains, if compared to the effect of soil and masonry stiffness variation.

4. Case study

In this section the relative stiffness method is applied to a case study from the London Underground Jubilee Line Extension. The aim is to evaluate the influence of (a) the structural assumptions and uncertainties in the building stiffness calculation and (b) the different formulations of the method on the final damage assessment of a real structure. The selected building is the Neptune House at Moodkee Street (Figure 18). This is a 3-storey load bearing masonry building affected by the construction of the two Jubilee Line tunnels and not subjected to any preliminary protective measure. The building, dated 1931, is approximately 40 m by 8 m in plan and has concrete strip footings [17]. Figure 19 shows the location of the 5 m diameter twin tunnels that are 17 m deep and were excavated in 1996 (first the Westbound –WB– tunnel and then the Eastbound –EB– 5 months later).

For twin tunnels, the location of the inflection point for the combined settlement trough induced by the two tunnels depends on the distance between the two tunnel axes. Assuming that the volume loss, the tunnel depth and the trough width parameter are similar for both tunnels, three general scenarios can be expected (Fig. 20, Crossrail [24]).

The Neptune House was mainly affected by the EB tunnel excavation. The two long façades, east and west, inclined at a 61 degree angle with respect to the EB tunnel axis, were subjected to a hogging deformation from the



Figure 18: Neptune House, view from the North West [23].

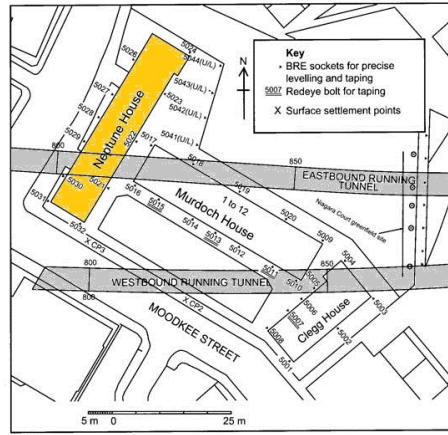


Figure 19: Plan of Neptune House and JLE running tunnels (after Mair and Taylor [17]).

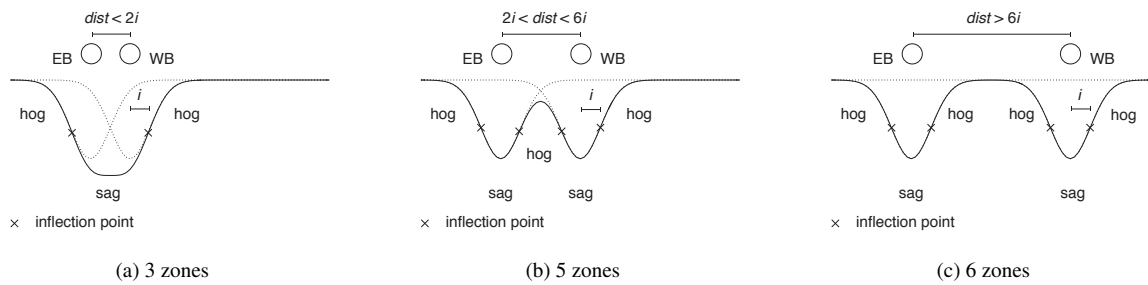


Figure 20: General trough shapes for twin tunnels (after [24]). The number of sagging (sag) and hogging (hog) zones depends on the distance ($dist$) between the Eastbound (EB) and the Westbound (WB) tunnel and on the distance (i) between each inflection point and the corresponding tunnel.

WB tunnel and to a combination of hogging and sagging deformations from the EB tunnel. Table 8 reports the observed deformation parameters [25].

Table 8: Deformation parameters for the Neptune House [25].

Façade	Angle to tunnel	Max differential	Max Δ_s/L_s	Max Δ_h/L_h
Façade	axis (deg)	settlement (mm)		
West façade – WB	58	2	–	3×10^{-5}
East façade – WB	58	3	–	4×10^{-5}
West façade – EB	61	4	4×10^{-5}	2×10^{-5}
East façade – EB	61	4	7×10^{-5}	3×10^{-5}

Mair and Taylor [17] gave an initial estimation of the building damage by assuming that the building response was governed by the masonry walls. Later, Dimmock and Mair [18] inferred the bending stiffness by the observed deformation parameters and related modification factors. Since the back calculated stiffness in hogging and sagging were 1 and 2 orders of magnitude lower than the predicted ones, respectively, Dimmock and Mair [18] reformulated the estimation by neglecting the contribution of the walls in hogging and by reducing the stiffness by one order of magnitude because of the windows.

This paper analyses in detail the impact of different assumptions related to the following factors:

- **Façade** The contribution of the façade to the global building stiffness can be either included or neglected.
- **Openings** The effect of window openings can be either neglected or taken into account by reducing EI by 90%.
- **Slab** The stiffness contribution of the slab can be either included or neglected.
- **Foundation** The foundation contribution can be included or neglected.
- **Neutral axis** The position of the neutral axis can be either assumed at half of the building height or calculated based on all element contributions to the global stiffness.

The different assumption combinations considered in this study are reported in Table 9. Case 2 represents the reference scenario, where the contribution of the façade is included, the stiffness is reduced by 40% to take the openings into account, both the ground floor slab and foundation are assumed to contribute to the building stiffness and the position of the neutral axis was calculated by including all primary structural elements.

Figure 21 shows the results in terms of bending stiffness for the east façade. The main contribution to the bending stiffness comes from the façade and the foundation strips, and can be largely dependent on the neutral axis position. Assuming the neutral axis at the mid-height of the structure increases the calculated stiffness by 42% (case 1), while neglecting the façade results in a negligible global stiffness (case 4). Without the façade, the neutral axis moves closer to the foundation, and therefore the foundation contribution to the bending stiffness is also significantly reduced.

Figure 22 shows the modification factors obtained for each of the assumed combinations by applying the RSMs. The monitored modification factors back-calculated from field monitoring data are 0.3 in sagging and 1 in hogging,

Table 9: Assumed combinations of structural features.

Case	1	2	3	4	5	6
Façade	X	X	X		X	X
Openings	X	X			X	
Slab	X	X	X	X		
Foundation	X	X	X	X		
Neutral axis	1/2 height	calculate	calculate	calculate	calculate	calculate

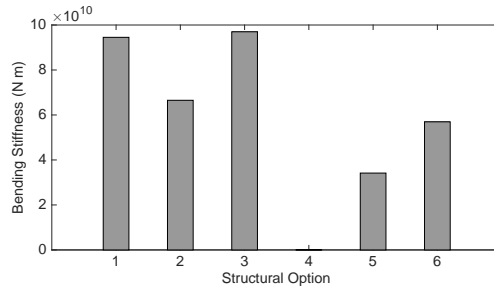


Figure 21: Neptune House case study: building stiffness for assumed combinations.

and are also plotted for comparison. As expected, the negligible global stiffness that results from ignoring the façade (case 4) leads to the highest modification factors for all formulations. At the other extreme, including the façade but ignoring openings (case 3), results in the lowest modification factors, both in sagging and in hogging. For the reference case (case 2), Potts and Addenbrooke and Goh & Mair's upper bound formulations give the best prediction for sagging, while Franzius' prediction is the closest to reality in the hogging case. Goh & Mair underpredict the response in hogging because the building partition method causes the length in the hogging zone to be relatively short, which predicts an unrealistically rigid structure. Goh & Mair's upper and lower bounds are the most sensitive to this variation. This sensitivity is mainly connected to the design curves. By increasing the relative bending stiffness, Goh & Mair's curves lead to a more rapid decrease in modification factors. For the specific sagging case analysed, $\rho_{\text{sag,par}}^*$ is high enough for Goh & Mair's lower bound to predict that the building is essentially fully rigid ($M^{\text{DR,sag}} \simeq 0$) unless the façade is completely ignored (case 4).

In general, the assessment given by Franzius et al. [5] is the most conservative, while the accuracy of the Goh and Mair predictions varied considerably whether the structure was in hogging or sagging, again because of the partitioning method employed. The same analysis has been performed on the west façade and by taking into account the effect of the EB tunnel only. The results do not differ significantly from the ones presented above.

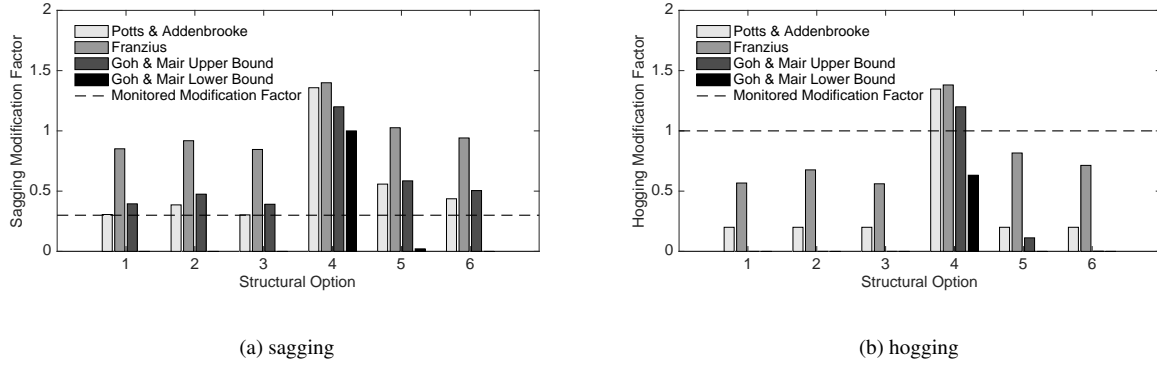


Figure 22: Neptune House case study: modification factors for assumed combinations.

5. Conclusions

This paper evaluated the available methods for the rapid assessment of settlement-induced damage to surface structures. In particular, it focused on the different formulations of the relative stiffness method, and on the available procedures for the calculation of building stiffness.

A sensitivity study performed on a number of structures by varying the building height-to-length ratio, eccentricity-to-trough width ratio, opening ratio, tunnel cover-to-diameter, soil and masonry stiffness, and trough width parameter made it possible to quantify the influence of these factors on the building stiffness calculation and final damage assessment. Results showed that the original RSM formulation by Potts and Addenbrooke [3] tends to give a prediction contained between the upper and lower curves by Goh and Mair [6], while the predictions provided by Franzius et al. [5] tends to be the most conservative.

In order to exemplify the actual effect of structural assumptions and different RTM formulations on damage predictions, these formulations were applied to a masonry building affected by the construction of the Jubilee Line in London. The results quantified the relative impact of the building façade, of window openings, and of the assumed neutral axis position on the global building stiffness calculation. Furthermore, they showed that the largest impact on the final assessment, apart from ignoring the façade entirely, is given by the RSM selection. These results provide information to guide engineers as they apply these approaches in practice, and information to aid the development of more robust and consistent procedures in the future.

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